Quarterly Progress Report #7

For the project entitled:

Field Evaluation of the Performance of Three Concrete Bridge Decks on Montana Route 243

Reporting Period: July 1, 2003 – September 30, 2003
(*Quarter 1, State Fiscal Year 2004*)

Summary of Expenditures

The table below summarizes the expenditures on this project through September 30, 2003. Expenditures during this quarter were $15,841.98, with total expenditures through September 30, 2003 equaling $227,789.30.

Budget Category	Spent through 9/30/03	Spent This Quarter	Total Spent
Salaries	$80,161.84	$14,875.84	$95,037.68
Benefits	$12,739.74	$2,849.05	$15,588.79
In-State Travel	$8,396.39	$4,979.44	$13,375.83
Expendable Supplies	$15,769.74	$622.30	$16,392.04
Tuition	$25,220.50	($12,150.00)	$13,070.50
Reporting	$0.00	$0.00	$0.00
MDT Direct Costs	$142,288.21	$11,176.63	$153,464.84
Overhead	$21,659.11	$4,665.35	$26,324.46
MDT Share	$163,947.32	$15,841.98	$179,789.30
WTI Share (Equipment and Out-of-State Travel)	$48,000.00	$0.00	$48,000.00
Total	**$211,947.32**	**$15,841.98**	**$227,789.30**

Over the past five months, it was critical to the success of the project to accomplish tasks associated with installing the sensors, monitoring construction of the bridge decks, conducting the live load tests and many other construction related activities. Most importantly, ensuring proper installation and wiring of the sensors was critical to the success of the project. Additional measures were taken to guarantee that this component of the project was done properly, since future adjustments or changes were impossible. Moreover, it was clear from preliminary meetings that the research project was to not to delay construction, if possible. Consequently, WTI was responsible for meeting all construction deadlines, which was successfully done.

The majority of the delay to the research tasks occurred after the bridge decks had been cast. Post construction activities proceeded slower than anticipated, making it difficult to coordinate research activities. Activities include deforming the bridge decks, paving the roadway, and placing the guardrail. These delays slowed installation of the conduit for the sensor leads, trenching for the power and communications cables, and running the live load tests.

The schedule in the original proposal anticipated that construction would begin in July and end by October 2002. Actual construction occurred between April and July 2003, pushing live load testing to the end of July/beginning of August (8 to 9 months later). A second series of live load tests is scheduled to occur two years after the first series, which will require that the end date of the project be extended. The second live load tests will occur in July/August of 2005. Moving the project end date to December 2005 will allow sufficient time to complete a final report after the final testing, thereby adding seven months onto the project. This schedule change will require additional funding, mainly to cover project management and reporting costs.

Task A: Project Management

Most of the activity related to project management focused on ensuring that research activities harmonized efficiently with scheduled construction events; namely, completing the plumbing and other miscellaneous activities, and conducting the live load testing. Significant effort was also spent on reviewing the project budget.

<u>Action Items for next quarter:</u>

- Meet with project technical panel to discuss the budget and timeline for the project

Task B: Conduct Literature Review

The primary literature review for this project has been completed. Nonetheless, the time frame for this project is quite long, so information will continue to be collected throughout its duration. During this quarter, significant literature was collected regarding live load testing of bridge structures. Knowledge gained during this search helped to finalize the live load test plans.

<u>Action Items for next quarter:</u>

- Continue collecting relevant literature
- Write up the literature review for the interim report

Task C: Develop Instrumentation Plan and Assemble Data Acquisition System

Determine Gage Locations

Information from the report documenting the process used to select the gage locations in each deck will be summarized and included in the interim project report.

Weather Station

A database was finalized to store the data received from the remote weather station. This database makes it possible to query for specific data, which will make subsequent analysis work more efficient. Weather data will be considered in conjunction with strain data collected from the bridges to help understand the effect of environmental factors on the behavior of the structures.

Bridge Monitoring Data Acquisition System

During this quarter, work on the data acquisition system generally focused on programming the data loggers for the live load tests and for capturing heavy vehicle events after the bridges were open to traffic. Note that the instrumentation for all three decks has been active since the time they were cast. For approximately ten days, strain and temperature readings from each gage were recorded every ten minutes to capture the rapid changes during construction and curing. Following this period, the interval between readings was lengthened to one hour. With the exception of brief interruptions for live load testing and system maintenance, the hourly schedule has persisted and is presently being maintained.

During the live load testing, all three data loggers were used collectively to monitor the response of a single bridge. The sensor configuration for long-term monitoring does not allow for rapid collection of data (>50Hz scan rate) due to limitations associated with the multiplexers. Therefore, to achieve the high speed data collection necessary during the live load tests, all three loggers were concurrently used. During the slow live load tests, a hand-held switch was connected to all three data loggers to synchronize measurements with truck position. Lines painted at 2-meter intervals on the deck were used to monitor the position of the truck as it traversed the bridge. The data collection computer program was also modified to accommodate four external strain gages from Bridge Diagnostics Incorporated (BDI) that were temporarily attached to the bottom of selected stringers during the live load tests.

Data was collected at a rate of 50Hz and 100Hz for the low- and high-speed tests, respectively. Reducing the rate for the slow tests resulted in smaller file sizes without sacrificing data quality.

Using the information from the live load tests, specific sensors were chosen to be live in order to capture strains associated with "large" vehicle events after the bridges were open to traffic. Selections were based on the quality and level of the output, which depended on their location and orientation. Accordingly, ten bonded strain gages and two embedded concrete strain gages were selected to capture deck response to large vehicle events. These gages were rewired to run through only two bridge completion boards to conserve battery power, and the long term data acquisition program was rewritten to include logging the data from these sensors when they are triggered by a large event. On each bridge, one of the 12 sensors is designated as a trigger gage to initiate the data storage process. The general magnitude of the response of these sensors was determined from the live load testing. Trigger values were set to be approximately 40% of the maximum response recorded for that sensor during the high speed live load tests. If the output from the trigger gage increases by more than 40% when a vehicle crosses the bridge, 1.5 seconds of data is captured before and after the event. A maximum of 60 events can be stored on the data acquisition computer before it needs to be downloaded and cleared. Information from these gages will be compared to data collected by the classifier and portable WIM for a one week period in September. These comparisons should allow for correlations to be made between the deck response and the class and weight of the vehicles that cause this response.

Action Items for Next Quarter:

- Complete historical documentation
- Preserve and maintain the accuracy of long-term monitoring system
- Maintain/download data from large vehicle events and compare to data from MDT portable WIM site

Task D: Install Instrumentation and Compile As-Built Documentation

Instrumentation Installation

WTI coordinated with Sletten Construction to finish deforming the Empirical and Conventional bridges. Following deforming, plumbing of the sensor wires into the data acquisition boxes was completed. All sensor wires are now fully plumbed to protect them from the surroundings. Permanent installation of the power and telecommunications cables still needs to be completed.

During the live load tests, four additional gages (Intelliducers ™ from Bridge Diagnostics Incorporated (BDI)) were temporarily affixed to the bottom of the stringers. These gages helped characterize the global response of the bridge. The gages were attached using quick-drying super glue. They were removed and attached to each structure as the live load tests were successively conducted on each bridge. A typical BDI gage and its installation are shown in Figure 1.

Figure 1: External Gage from Bridge Diagnostics Incorporated.

Action items for next quarter:

- Conduit communication and power cables from the power pole to the data acquisition box

Materials Testing

Tests were conducted on the concrete specimens cast during bridge construction to determine the properties of the deck concrete at the time of the live load tests. Tests were performed to determine the unconfined compression strength, splitting tensile strength, and modulus of rupture of the concrete. A summary of the samples collected from each bridge, the conditions under which these samples were or continue to be cured, and the tests to which these samples have or will be subjected is presented in Tables 1 through 3. Specimens to be cured with the bridges were stripped at approximately the same time as the bridges were deformed. These specimens presently are being cured in the MDT maintenance yard in Saco.

Twenty-eight day compression tests were performed on moist cured samples from the Conventional deck. Load and displacement data was collected from these compression specimens. Results from these tests are summarized in Table 3.

During bridge construction, MDT performed air content tests, conducted slump tests, and collected compression test specimens from the concrete from selected trucks on each bridge. Results of these tests will be included in the project analysis and documentation.

Action Items for Next Quarter

- Continued measurement of shrinkage specimens
- Collect material properties of the reinforcing steel and concrete from MDT

Table 1: Concrete Sampling and Testing Matrix – HPC Deck, Cast 5/28/03.

Truck No.	Type of Specimen[a]	Number of Specimens	Curing	Time to be tested	Type of test	Results
H-4	cylinder	1	moist	28 days	compression	49.9 MPa
	cylinder	2	with deck	1st live load	compression	54.6 MPa
	cylinder	2	with deck	1st live load	split-cylinder	4.0 MPa
	cylinder	2	with deck	2nd live load	compression	$\sigma - \varepsilon$ curve
	Beam	2	with deck	1st live load	bending	4.2 MPa
	Beam	1	with deck	2nd live load	bending	
	Shrink	3	with deck	periodically	shrinkage	
	Shrink	3	moist	periodically	shrinkage	
H-6	cylinder	2	moist	28 days	compression	45.6 MPa
	cylinder	2	with deck	1st live load	compression	46.2 MPa
	cylinder	2	with deck	2nd live load	compression	$\sigma - \varepsilon$ curve
	Beam	2	with deck	1st live load	bending	4.2 MPa
	Beam	1	with deck	2nd live load	bending	
H-8	cylinder	1	moist	28 days	compression	43.0 MPa
	cylinder	2	with deck	1st live load	compression	46.9 MPa
	cylinder	2	with deck	2nd live load	compression	$\sigma - \varepsilon$ curve
	Beam	1	with deck	1st live load	bending	3.9 MPa
	Beam	2	with deck	2nd live load	bending	

[a] *cylinder – 152 mm (6 in) diameter by 305 mm (12 in) long cylinder*
beam – 152 mm (6 in) wide by 152 mm (6 in) deep by 508 mm (20 in) long beam
shrink – 76 mm (3 in) wide by 76 mm (3 in) deep by 406 mm (16 in) long beam

Table 2: Concrete Sampling and Testing Matrix – Empirical Deck, Cast 6/02/03.

Truck No.	Type of Specimen[a]	Number of Specimens	Curing	Time to be Tested	Type of test	Results
E-5	cylinder	3	moist	28 days	compression	27.7 MPa
	cylinder	3	with deck	1st live load	compression	32.6 MPa
	cylinder	3	with deck	2nd live load	compression	σ – ε curve
	Beam	2	with deck	1st live load	bending	3.3 MPa
E-7	cylinder	3	moist	28 days	compression	27.5 MPa
	cylinder	3	with deck	1st live load	compression	32.0 MPa
	cylinder	3	with deck	1st live load	split cylinder	3.0 MPa
	cylinder	3	with deck	2nd live load	compression	σ – ε curve
	cylinder	3	with deck	2nd live load	split cylinder	
	Beam	2	with deck	1st live load	bending	3.3 MPa
	Beam	2	with deck	2nd live load	bending	
	Shrink	3	with deck	periodically	shrinkage	
	Shrink	3	moist	periodically	shrinkage	
E-9	cylinder	3	moist	28 days	compression	27.0 MPa
	cylinder	3	with deck	1st live load	compression	34.1 MPa
	cylinder	3	with deck	2nd live load	compression	σ – ε curve
	Beam	2	with deck	2nd live load	bending	

[a] *cylinder – 152 mm (6 in) diameter by 305 mm (12 in) long cylinder*
beam – 152 mm (6 in) wide by 152 mm (6 in) deep by 508 mm (20 in) long beam
shrink – 76 mm (3 in) wide by 76 mm (3 in) deep by 406 mm (16 in) long beam

Table 3: Concrete Sampling and Testing Matrix – Conventional Deck, Cast 6/05/03.

Truck No.	Type of Specimen[a]	Number of Specimens	Curing	Time to be tested	Type of test	Results
C-4	cylinder	3	moist	28 days	compression	33.4 MPa
	cylinder	3	with deck	1st live load	compression	41.0 MPa
	cylinder	3	with deck	2nd live load	compression	σ – ε curve
	Beam	2	with deck	1st live load	bending	3.3 MPa
C-5	cylinder	2	with deck	1st live load	compression	34.7 MPa
	cylinder	2	with deck	2nd live load	compression	σ – ε curve
C-6	cylinder	3	moist	28 days	compression	32.1 MPa
	cylinder	3	with deck	1st live load	compression	37.5 MPa
	cylinder	3	with deck	1st live load	split cylinder	5.7 MPa
	cylinder	3	with deck	2nd live load	compression	σ – ε curve
	cylinder	3	with deck	2nd live load	split cylinder	
	Beam	2	with deck	1st live load	bending	3.7 MPa
	Beam	2	with deck	2nd live load	bending	
	Shrink	3	with deck	periodically	shrinkage	
	Shrink	3	moist	periodically	shrinkage	
C-7	cylinder	3	moist	28 days	compression	29.4 MPa
	cylinder	3	with deck	1st live load	split cylinder	5.6 MPa
	cylinder	3	with deck	2nd live load	compression	σ – ε curve
	Beam	2	with deck	2nd live load	bending	

[a] *cylinder – 152 mm (6 in) diameter by 305 mm (12 in) long cylinder*
beam – 152 mm (6 in) wide by 152 mm (6 in) deep by 508 mm (20 in) long beam
shrink – 76 mm (3 in) wide by 76 mm (3 in) deep by 406 mm (16 in) long beam

Task E: Live Load Testing

Live load tests were conducted on the three bridges the week of July 28. Testing got underway the afternoon of Tuesday, July 29, and all tests were completed by noon on Friday, August 1. The live load tests were conducted using two 3 axle dump trucks provided by MDT. The weights and dimensions of these trucks are summarized in Figures 2 and 3, respectively. For each bridge, the planned low speed single truck tests were done first, followed by low speed two truck tests and finally by high speed single truck tests. In the low speed single truck tests, one of the dump trucks was driven across the bridge several times at a speed of approximately 3 miles per hour. In each run, the truck was guided along one of eight of the longitudinal paths shown in Figure 4 (lines R, S, T, U, V, X, Y, and Z). In the low speed two truck tests, the two dump trucks were driven across the bridges side-by-side at a speed of approximately 3 miles per hour, along two longitudinal paths (side-by-side on lines W and R, and on lines X and T – Figure 4). Finally, in the high speed single truck tests, one of the dump trucks was driven across the bridge at a speed of approximately 60 miles per hour along one of five longitudinal paths (lines S, U, V, X, and Y – Figure 4).

Analysis of the data collected during the live load tests began immediately after the tests were completed. Steps in the analysis process consisted of assembling the data collected by the individual loggers, interpolating this data, as necessary, to a common position axis, and shifting the data from ambient strain levels (resulting from shrinkage and temperature effects) to a zero baseline. As described under Task C, the three individual data loggers were used on a single bridge during the live load experiments to be able to rapidly and simultaneously collect strain data from all reinforcement and external strain gages. This setup yielded three unique data files for each test run. Minor inconsistencies inherent with the three data loggers resulted in different data file sizes and data spacing ,thereby complicating future data processing. For inter-bridge comparisons, equal data spacing and length was desirable. As such, data manipulation was necessary to resolve these issues.

During the low speed live load tests a handheld switch was used to mark the position of the front axle with respect to equally spaced transverse lines painted on the bridge deck. These transverse lines spaced at two-meter intervals began with the north edge of the bridge deck. The handheld switch was connected in parallel to each of the data loggers so that position of the truck would be recorded as it traversed the bridge. Using this information, time and spatial information could be synchronized using the initial button-click when the front axle was at the edge of the bridge. The lengths of the individual data sets, however, were not the same, making it difficult to simultaneously compare data. To adjust these differences, strain values were interpolated to correspond with 2 cm intervals along the bridge deck. The results of these two techniques yielded data sets for a particular live load experiment that began at the same point in time and contained strain information for equally spaced intervals.

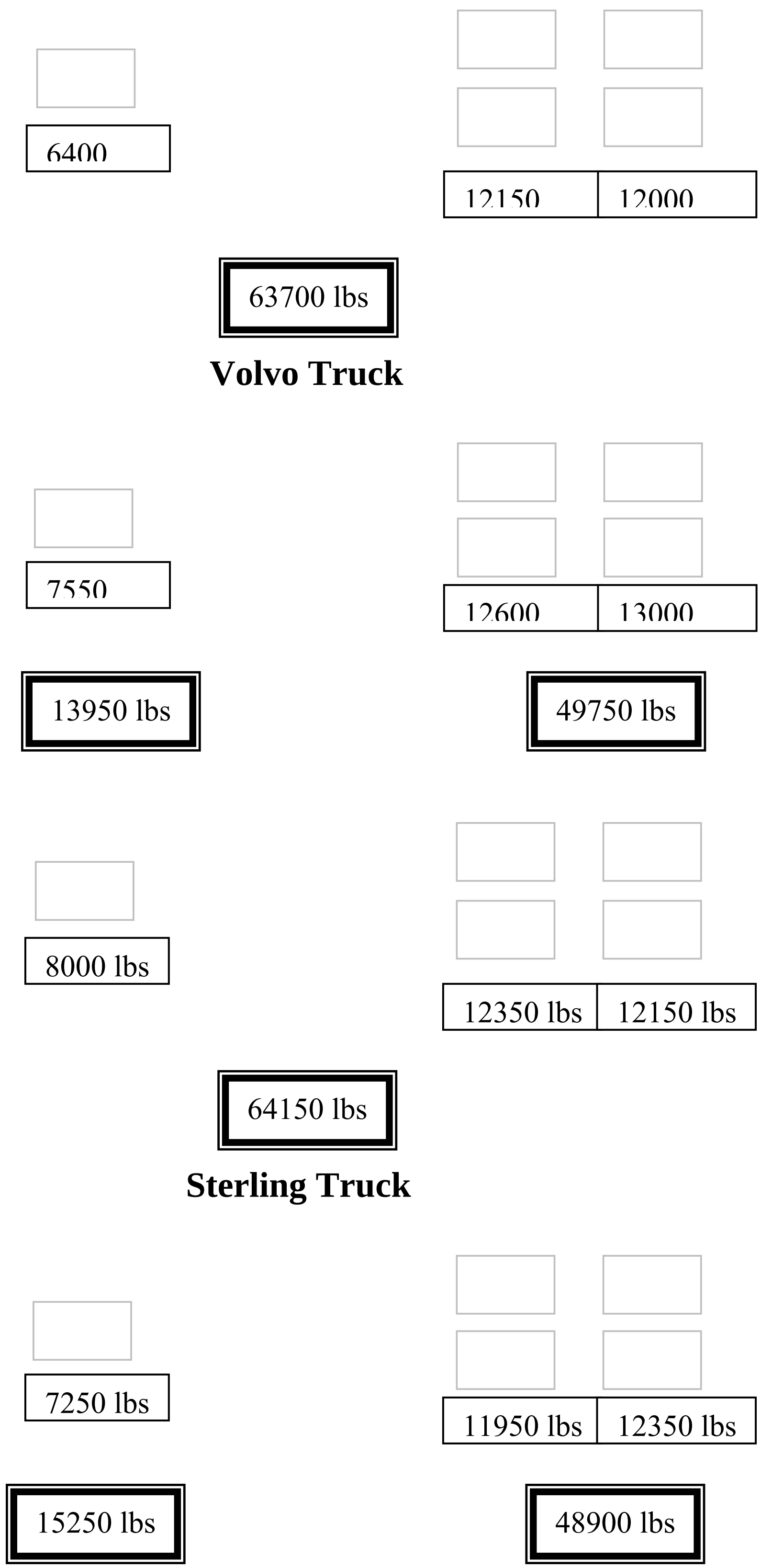

Figure 2: Weights of 3-Axle Dump Trucks Used during Live Load Tests.

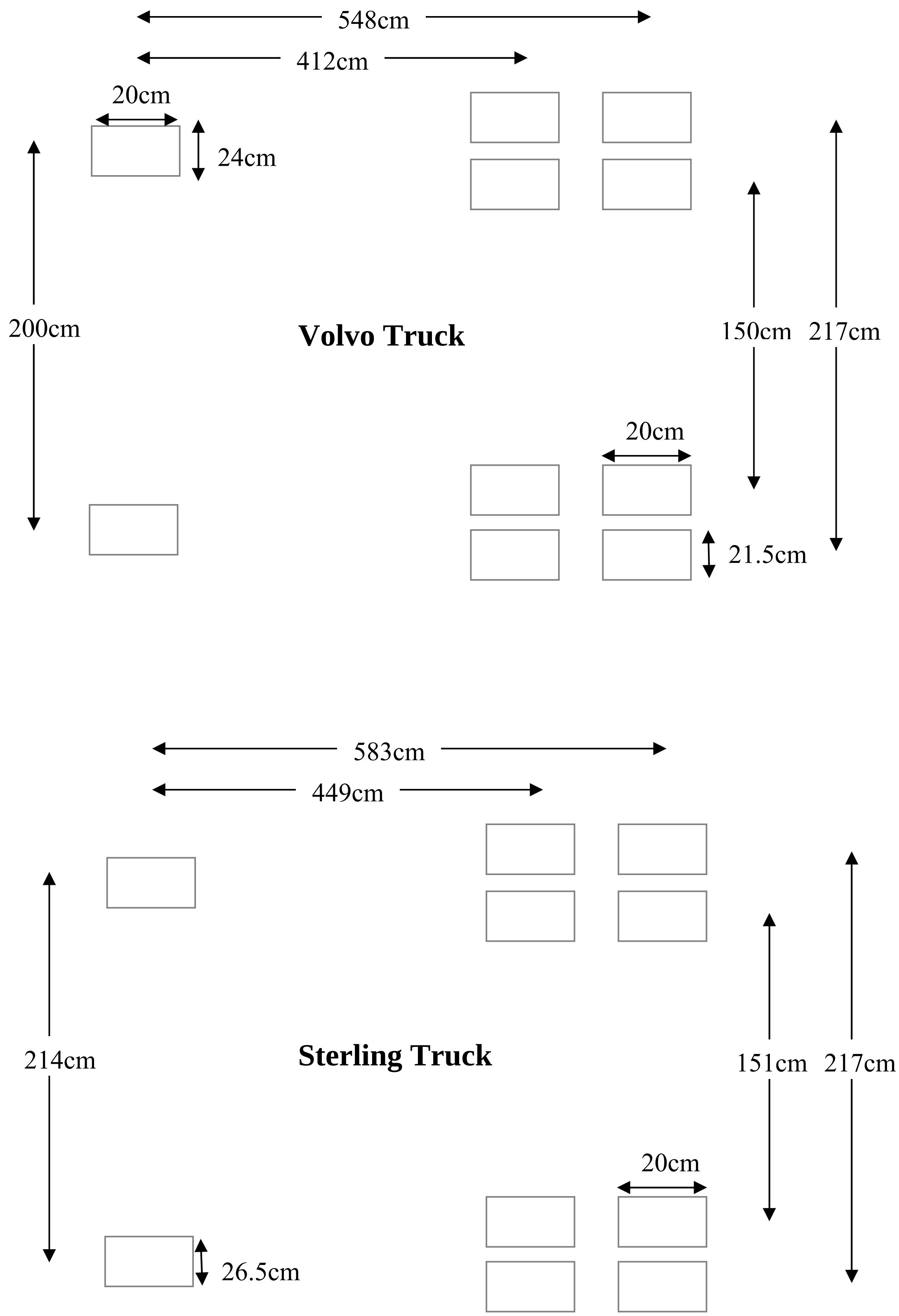

Figure 3: Dimensions of 3-Axle Dump Trucks Used during Live Load Tests.

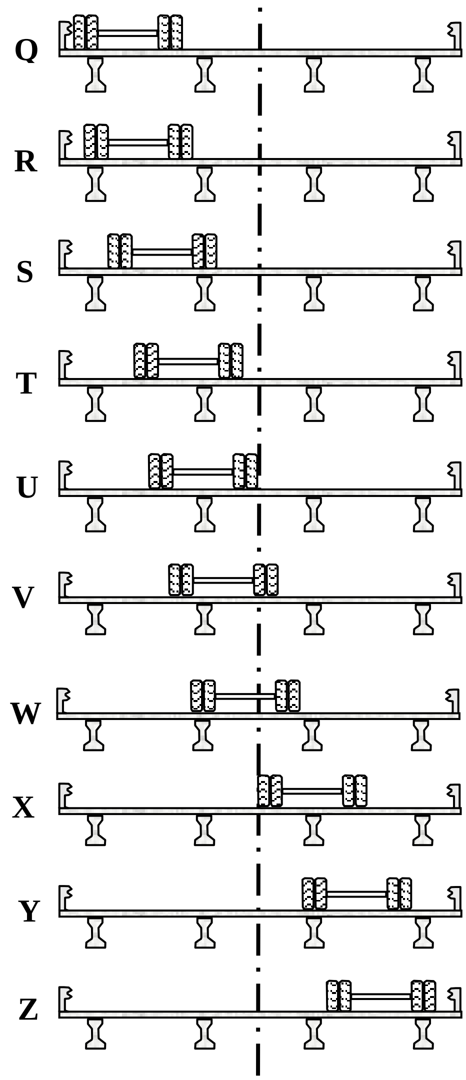

Figure 4: Truck Positions for Live Load Tests.

Once the various data manipulations described above were completed, work began on analyzing the live load test data relative to the structural behavior of the three bridges. Selected data and some preliminary observations on what it indicates relative to structural behavior are presented below. This phase of the data analysis effort is just getting underway, and significant time will be spent on this task during the next reporting period. All of the data presented below is for the response at a fixed point in the bridge as the test vehicle traversed the structure. Thus, these traces are similar to structural influence lines, in which the single moving concentrated load used in influence line construction has been replaced by a fixed sequence of three moving concentrated loads (the three axles of the test vehicle). The plots specifically show the response at the various gage locations as a function of the position of the steer axle of the test vehicle. The origin of the position axis for these plots is at the beginning of the bridge (0 m). In viewing this presentation of the data, other positions of interest include the locations of the bents (at 15 m and 30 m), the end of the bridge (at 45 m), and the position of the truck when the back axles leave the bridge (at 53 m).

Transverse Strain Response in the Deck

Typical deck response in the transverse direction during a low speed single vehicle test is shown in Figure 5 (results for the conventional deck are shown in the figure). This figure presents the transverse strains measured in the top and bottom of the slab at approximately the 1/3 point between the stringers for a test in which the truck was driven across the bridge with the driver's side wheel line directly over the first interior stringer and the passenger's side wheel line near the 1/3 point along the span between the stringers (as shown in Figure 5). Thus, the passenger's side wheel line passed directly over this gage location. Referring to Figure 5, the top and bottom of the deck experienced compression and tension, respectively, as the wheel loads passed. Passage of the individual axles of the test vehicle is clearly evident in the three discrete peaks in the waveforms (corresponding to the single steer axle followed by the two axles in the tandem). As might be expected, the response indicates positive bending moment occurred in the slab as the deck transferred the wheel loads transversely to the adjacent stringers. Maximum strains of 12 and 28 microstrain in compression and tension, respectively, occurred at this location in the deck. The difference in the magnitudes of the compression and tension strains is under investigation, as it indicates the cross-section is either non-symmetric or is experiencing a net axial strain. This difference could result from either tensile cracking in the cross-section (not expected at these low strain levels, at least from short term load effects), or the presence of net tension across the depth of the cross-section. Behaviors of this kind, that is, behaviors whose source is not obvious, will continue to be investigated as the analysis proceeds. The long term data will also be consulted for evidence of changes in bridge conditions that influence live load response (e.g., occurrence and growth of shrinkage/temperature cracks).

Figure 6 shows the deck response in the transverse direction at various locations in the deck for the same vehicle event described in the paragraph above. In this case, transverse strains are presented for the bottom of the deck at four locations along the same transverse section (as shown in the figure). Consistent with maximum concentrated load effects occurring at the point of load application, the highest strains were recorded by the gage almost immediately below the wheel line (C-TR-D-2-B). As would be expected, responses at the other three locations in the deck steadily decreased (relative to that directly under the wheel line), approximately in proportion to their distance from the passenger's side wheel line.

Transverse deck strains measured in all three bridges (conventional (CON), empirical (EMP), and high performance concrete (HPC)) at the same location for the same test run are presented in Figure 7. The particular data shown are again for a location at the 1/3 point in the span between the stringers, for a test run with the driver's side wheel line over a stringer and the passenger's side wheel line over the gage location. For this location and test run, the strains are similar in magnitude and waveform for all three bridge types. Notably, the strains in the empirical deck are not obviously different the strains in the conventional or HPC decks. Significant additional performance comparisons need to be made before trends of this type can be credibly established.

Longitudinal Strain Response in the Deck

Typical deck response in the longitudinal direction during a low speed single vehicle test is shown in Figure 8 (results for the conventional deck are shown). The strains measured in the top and bottom of the deck at a location at the 1/3 point between the stringers is shown in this figure for the same test run used above in discussing the transverse strain response (driver's side wheel line over the interior stringer, passenger's side wheel line at the 1/3 point of the span between the stringers). Referring to Figure 8, the strains in the top and bottom of the slab are similar in magnitude and sense (tension or compression) throughout the test event. These results indicate that as would be expected, the slab acted with the stringers to form a T-beam, with the neutral axis of the T-beam apparently dropping down into the "web" of the composite cross-section. Thus, the strains in the top and bottom of the deck were of the same sense, with the strains in the more extreme fiber in the top of the deck being nominally larger in magnitude than the strains in the bottom of the deck at a location closer to the neutral axis of the composite cross-section. Furthermore, this data may indicate the presence of limited continuity effects between the 15 m spans of the bridge. When the truck was exclusively in the first span (0 to 15 m), the response in the third span (at the gage location in the top of the deck) was nominally compressive, consistent with alternate span deflection effects in beams that are continuous over supports. Additionally, when the truck moved onto and off of the second span (around 15 m to 30m, respectively), the response in the adjacent third span (at the gage location in the top of the deck) was nominally tensile, once again consistent with adjacent span deflection effects in continuous beams. As the test vehicle entered the third span (the span in which the response was being measured), pronounced compression strains were observed, peaking at 33 micro-strain when the tandem axle was immediately in the vicinity of the gage location.

The longitudinal strains measured at a single location (the same location discussed above) for a sequence of low speed test runs are reported in Figure 9. As would be expected, as the longitudinal path of the test vehicle was successively moved away from the instrumented location (moving from path S to path Z), the response at the instrumented location decreased. Thus, these results provide an indication of the transverse distribution of applied loads across the various stringers in the bridge (and should be relatable to the AASHTO distribution factor for this superstructure arrangement).

The longitudinal strains measured at the same location in all three bridge decks for the same test run are shown in Figure 10. As observed for the transverse strain response, the responses of the three types of decks at this location are similar in magnitude and waveform. In this case, the maximum compression strain measured in the HPC deck is nominally smaller than that measured in the conventional and empirical decks. The HPC deck is made of substantially stronger concrete than the conventional and empirical decks, which should result in this deck having stiffer concrete and lower strains than the other two decks. Once again, however, before steadfast conclusions of this type are drawn from the data, a considerably more detailed review must be completed.

High Speed Tests

Typical longitudinal strains measured at the same location for the same test vehicle traveling at low and high speeds on the same longitudinal path are presented in Figure 11 (results for the conventional deck are shown). For this vehicle path and in this span, the maximum strain reported in the high speed event was only 90 percent of the maximum strain reported in the low speed event. Generally, the dynamic load allowance used for design is greater than one (1.33). This allowance is significantly influenced by surface and approach roughness. With new, smooth bridge decks and approaches, a value for this factor approaching unity seems reasonable. In this case, the further apparent reduction in strain in the high speed test relative to the low speed test speed may possibly be explained by vehicle positioning error. Vehicle position was more difficult to control in the high speed tests relative to the low speed tests. Thus, the longitudinal path of the vehicle may not have been identical in the two tests. Therefore, until further review of the data is completed, care should be exercised in attaching too much significance to the difference in strain response observed above.

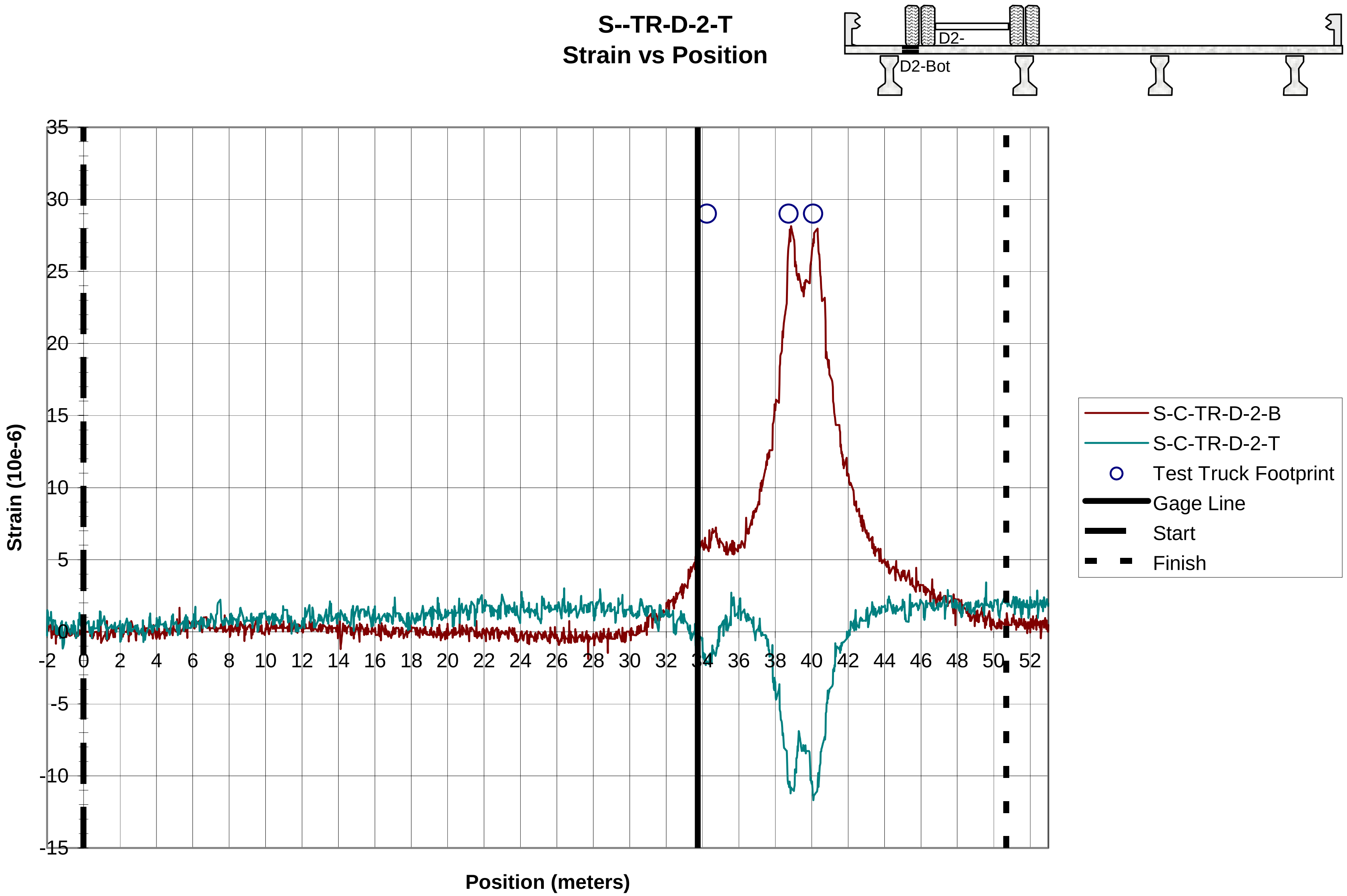

Figure 5: Transverse Strain Response at a Point in the Deck, Low Speed Test (Conventional deck shown).

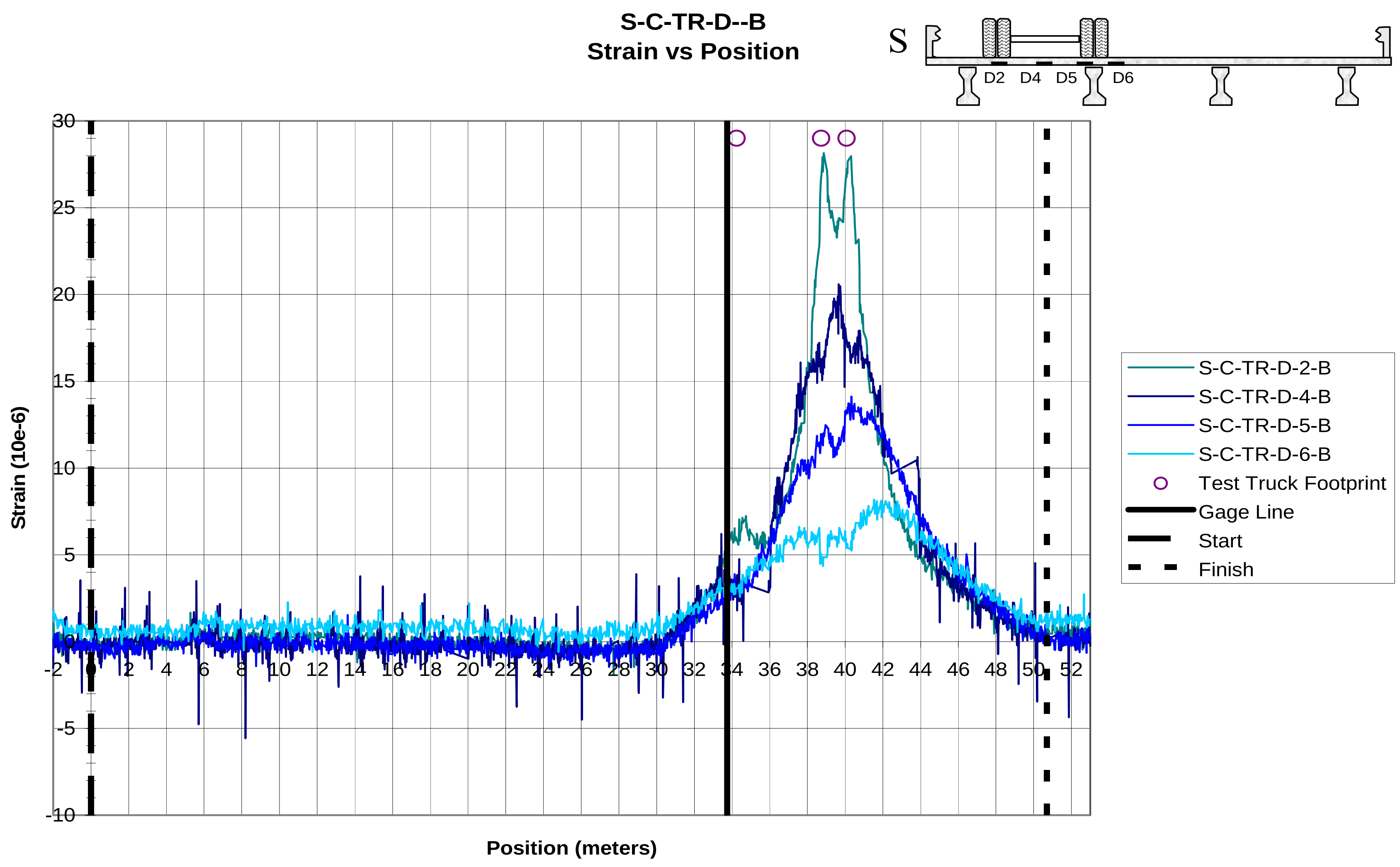

Figure 6: Transverse Strain Response Along a Cross-section in the Deck, Low Speed Test (Conventional deck shown).

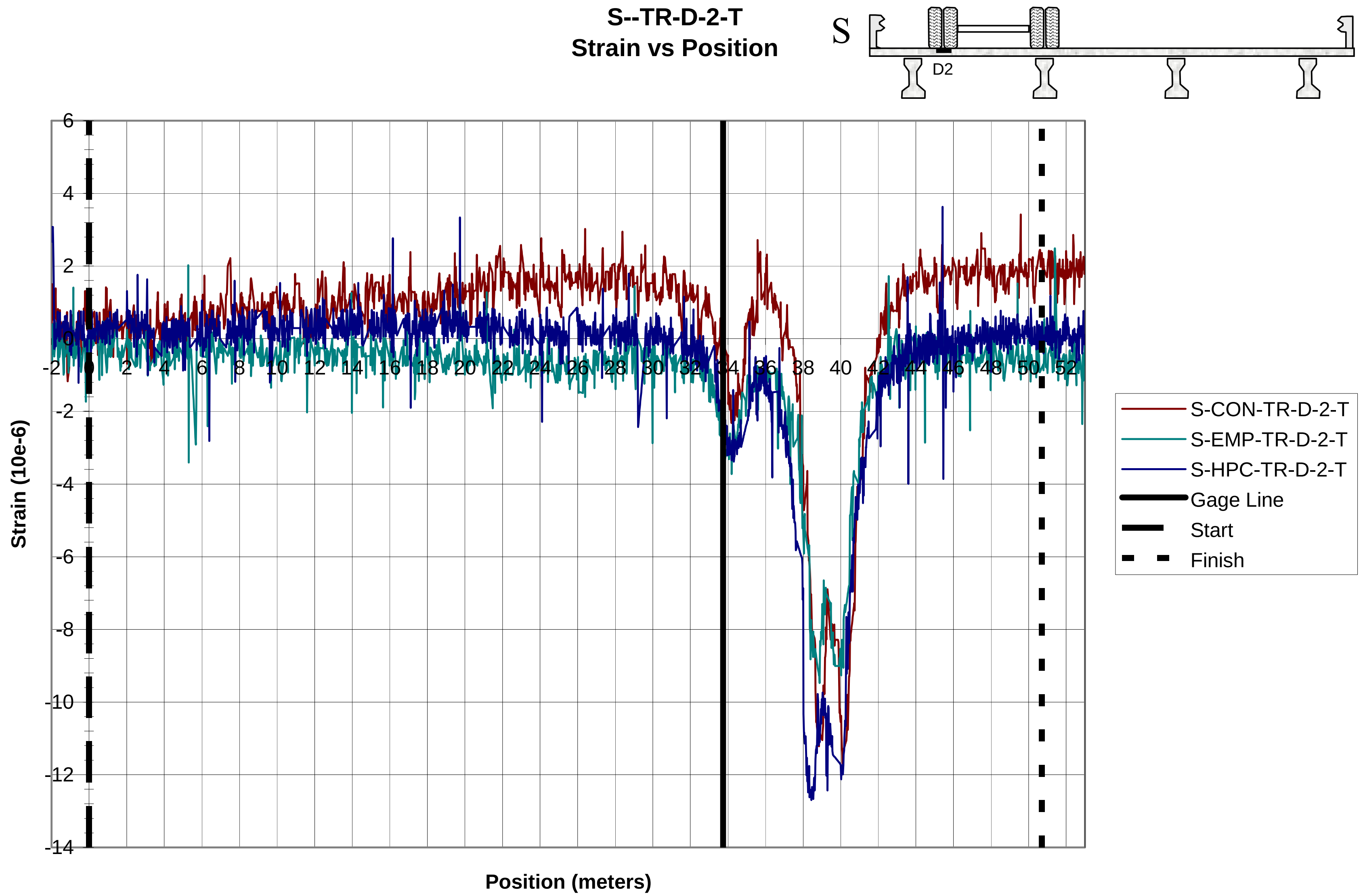

Figure 7: Comparison of the Transverse Strain Response of the Conventional, Empirical, and HPC Decks, Low Speed Test.

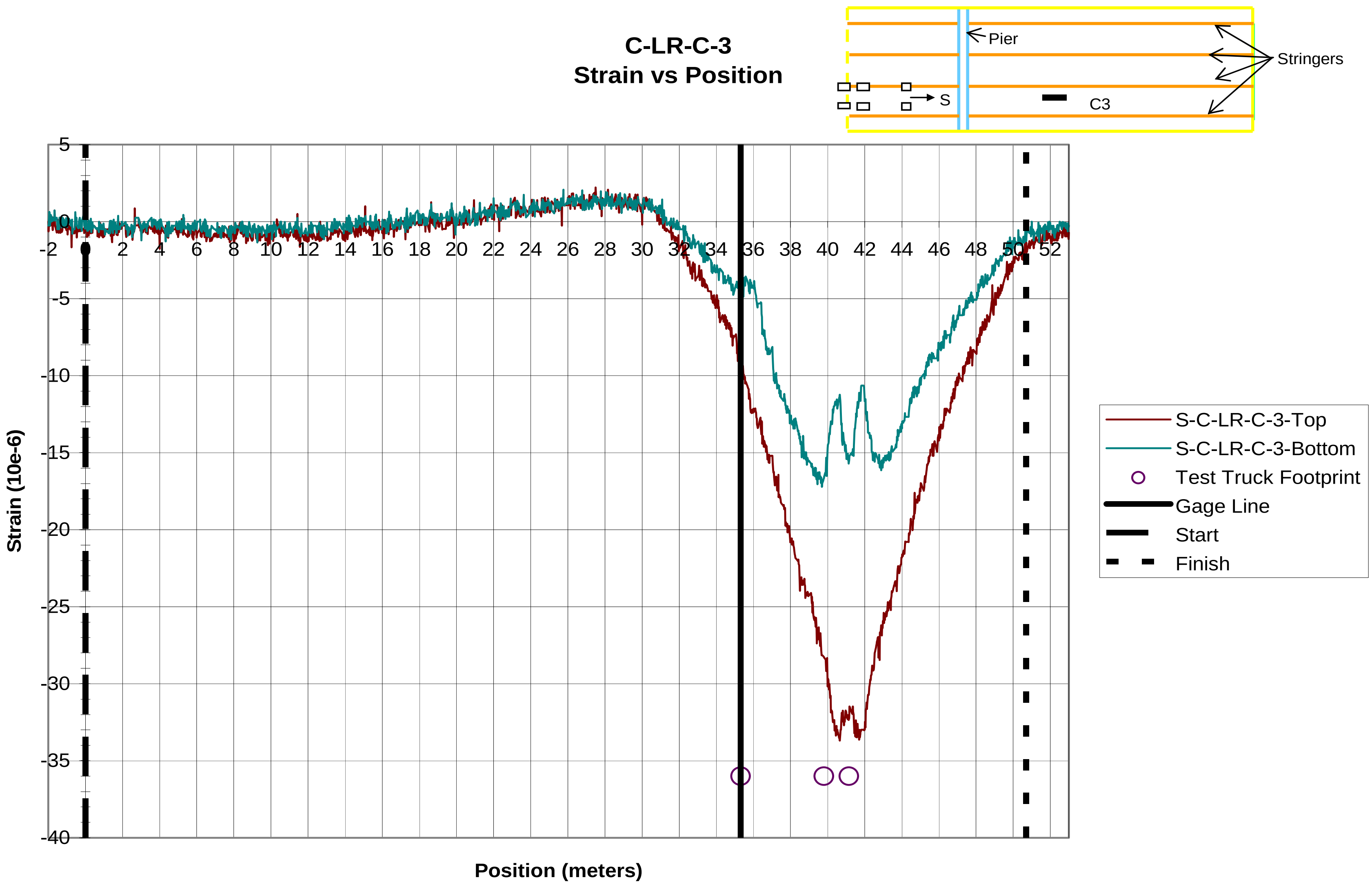

Figure 8: Longitudinal Strain Response at a Point in the Deck, Low Speed Test (Conventional deck shown).

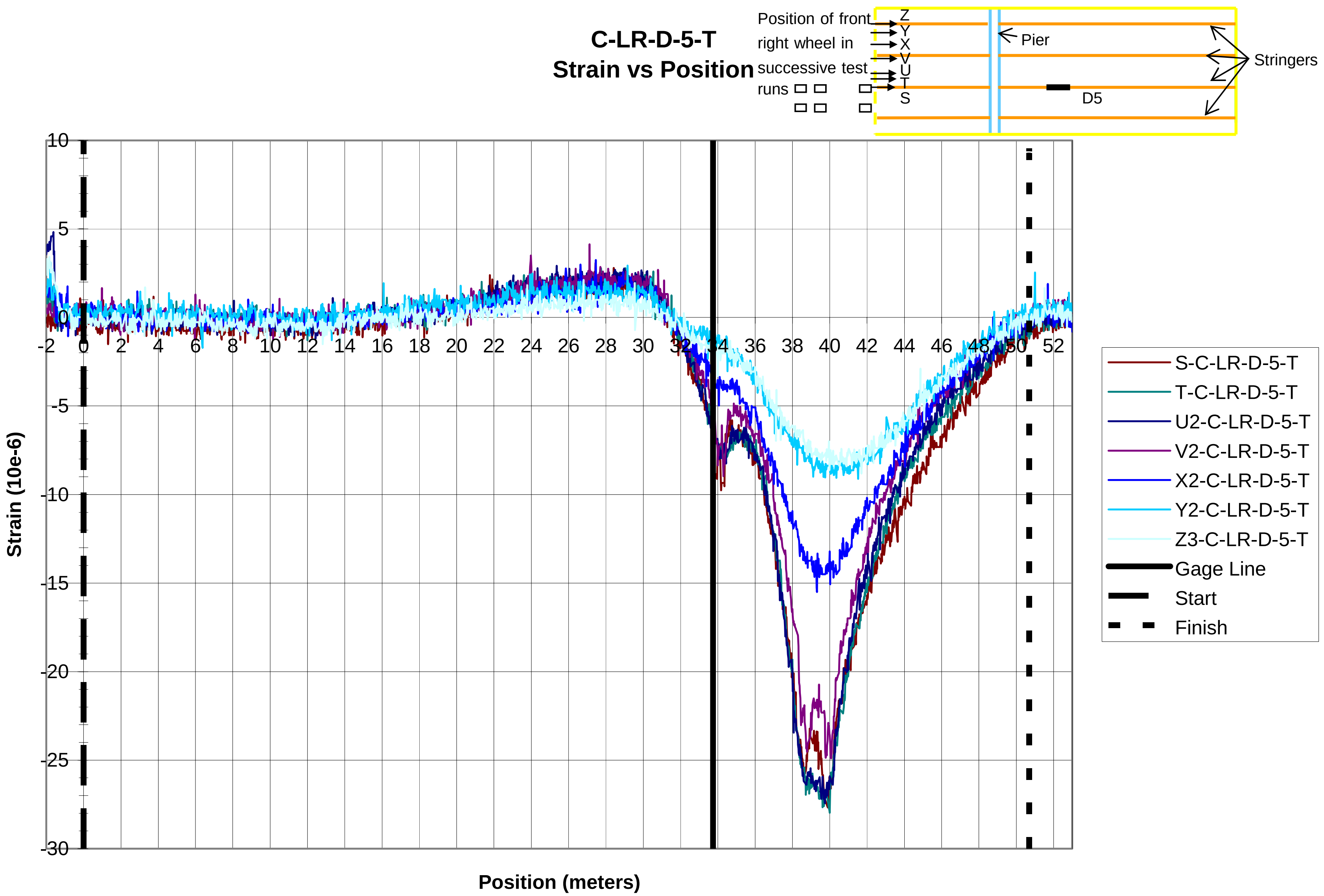

Figure 9: Longitudinal Strain Response at a Point in the Deck for a Progression of Low Speed Tests (Conventional deck shown).

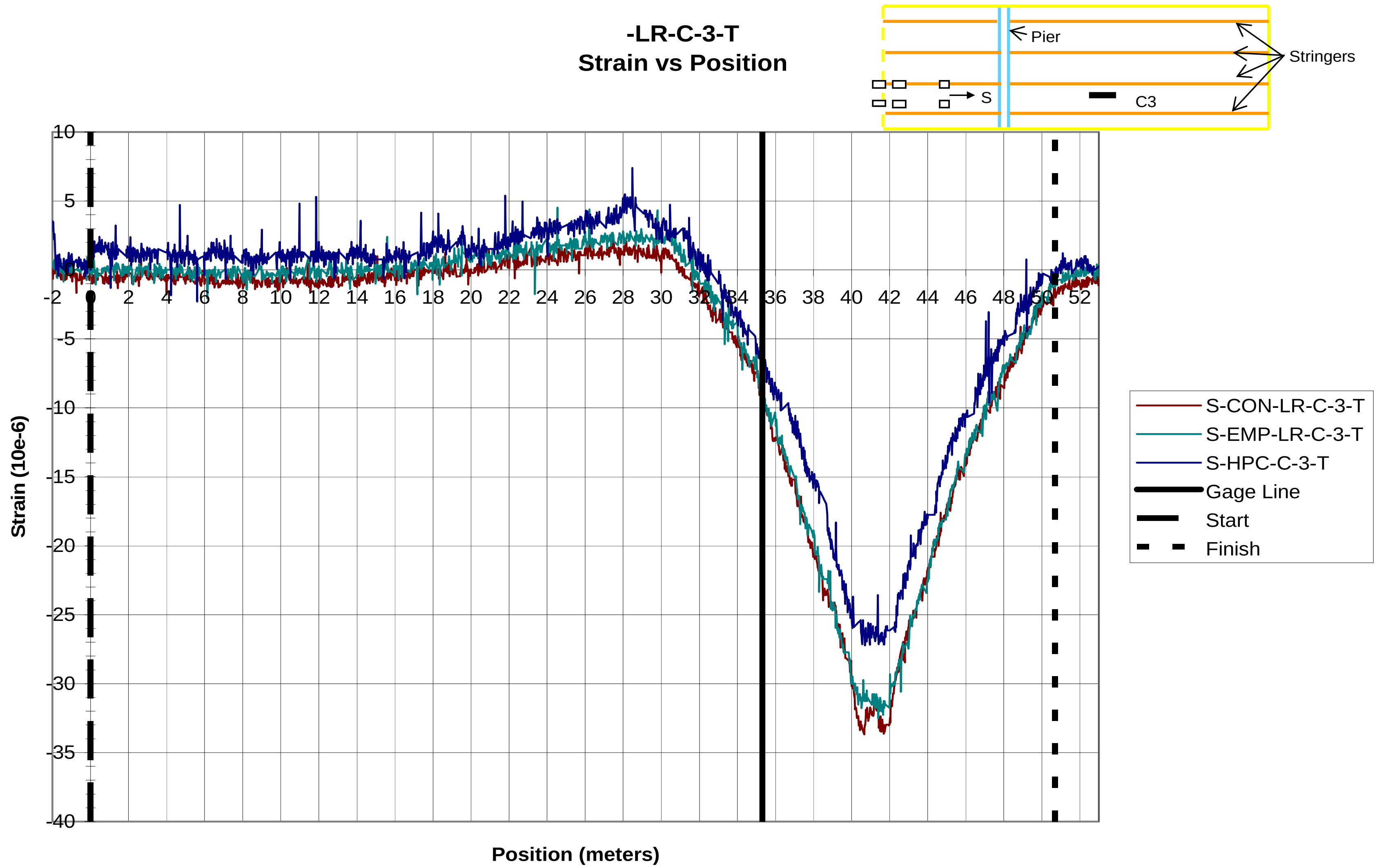

Figure 10: Comparison of the Longitudinal Strain Response of the Conventional, Empirical, and HPC Decks, Low Speed Test.

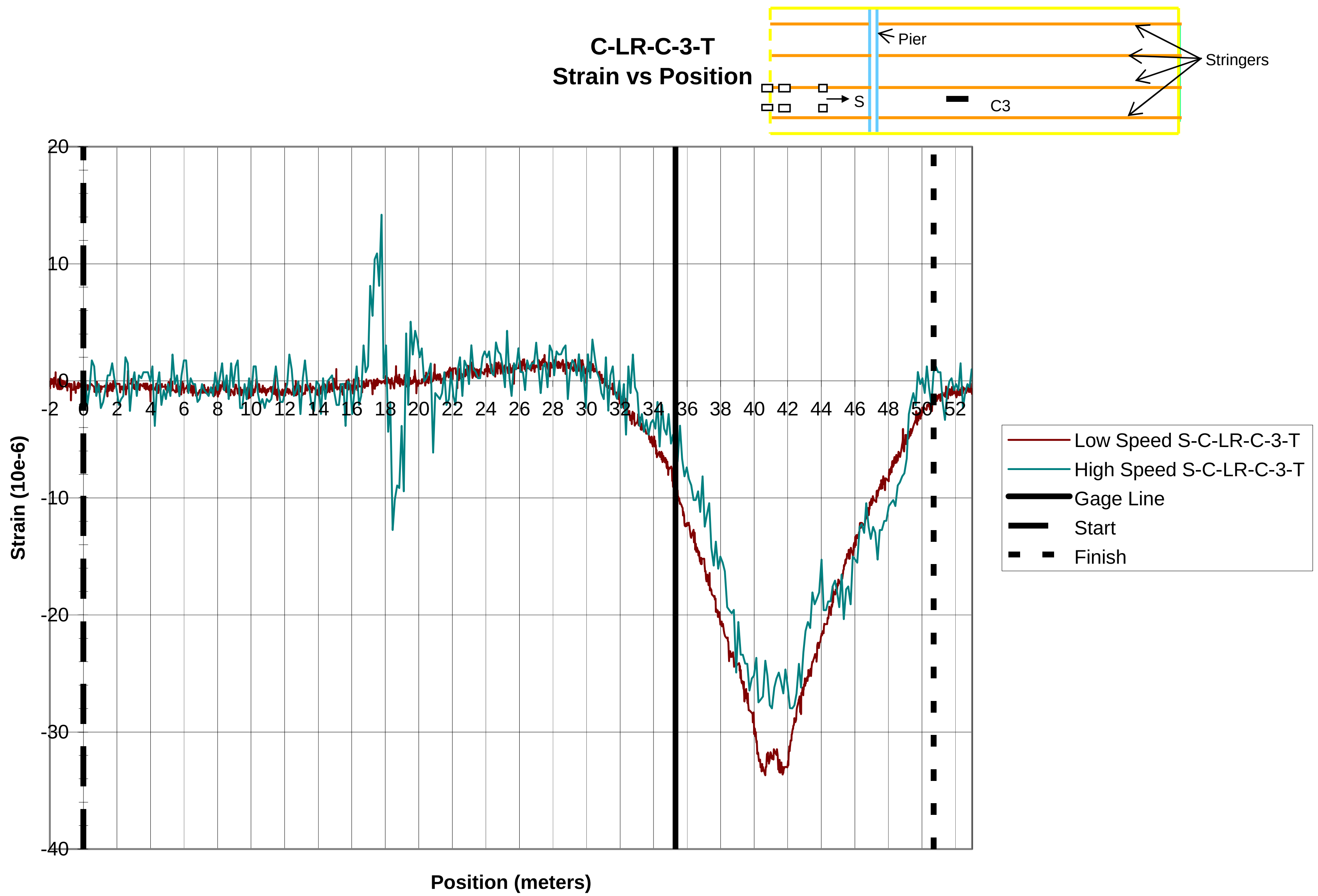

Figure 11: Comparison of the Longitudinal Strain Response in Low and High Speed Tests (Conventional deck shown).

Action Items for Next Quarter:

- Continue analysis of live load test data

Task F: Long-Term Monitoring

Strain Monitoring

Approximately four months of long-term data has been collected from embedded sensors in each of the bridge decks. Initially, 35 strain gages were installed on the steel reinforcement, and 9 concrete strain gages and 20 vibrating wire strain gages were embedded in the concrete. The data acquisition system was able to accommodate all 35 strain gages on the reinforcement, 7 of the concrete strain gages and 16 of the vibrating wire strain gages. Since their installation, some of the gages have ceased to function, and idle gages have been connected to replace the unresponsive ones. Gages registering higher strains, like those spanning the bents, are more susceptible to damage from these higher strains, notably due to cracking. On all three decks, several of the embedded concrete strain gages have stopped working properly for unknown reasons. Efforts to determine the reason for these failures are ongoing.

All the active long term sensors are currently set up to provide measurements once every hour. This data acquisition schedule has been interrupted on occasion due to maintenance and other activities. These interruptions are considered inconsequential in terms of the entire length of the project and are noticed as gaps in the graphs of the long term data.

The data available from the long term monitoring effort will be studied over the next several months with the goal of correlating changes in deck performance with the vehicle and environmental loads they experience, and then to further evaluate the relative performance of the three types of decks. Presented herein are some early (and preliminary) observations from this process. Notably, specific examples of long-term effects that clearly illustrate data trends, crack formation and shrinkage are considered. Several sensors were installed over the interior bents (mostly Bent #2) to capture the effects of anticipated cracking. Physical cracks have been observed over all the interior bents, and the data collected from the vibrating wire strain gages over Bent #2 (gage-line F) reflect the occurrence of these cracks. Figure 12 shows the physical location of the gages considered in this preliminary review of long term data. In general, cracking that intercepts the vibrating wire gage will increase cyclic strain levels due to daily and seasonal temperature fluctuations. The gage length of the vibrating wire strain gages is 153 mm, so prior to the formation of a crack anywhere along its length, measured strains would indicate tensile or compressive strains only in this region. After a crack forms, strains will also include differential movements of the separate concrete segments. Figure 13 illustrates this phenomenon using strain data from vibrating wires on gage-line F for all three decks. Recall that the HPC bridge deck was poured on May 28, the Empirical deck on June 2 and the Conventional deck on June 5. Both the HPC and Conventional decks show small daily strain fluctuations (~10-20με) prior to cracking. Once the crack formed, daily fluctuations are greatly increased to approximately 250με. The empirical deck behaved slightly differently in that the data indicated the presence of a hairline crack in the deck shortly after the pour and a more fully developed crack occurring later. Notably, all cracks seem to have either formed or fully formed near June 19th. Further investigation into this matter may reveal the reason for this concurrent behavior.

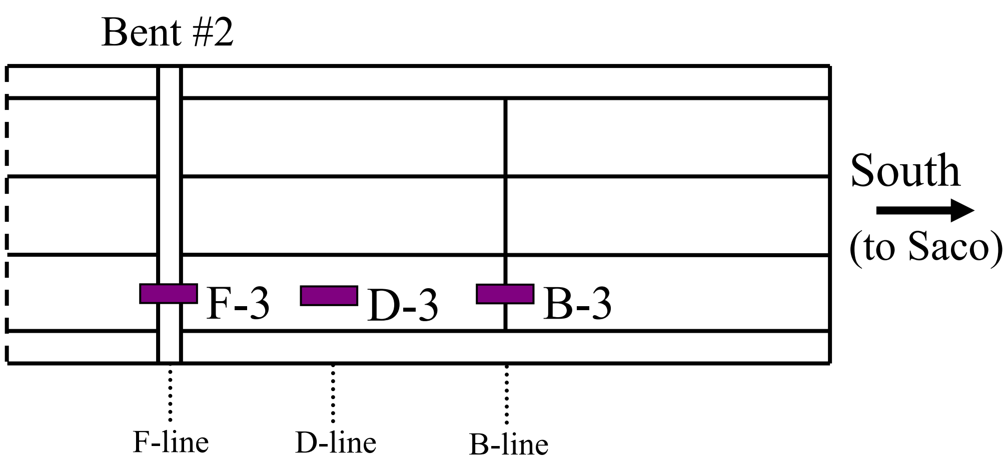

Figure 12: Vibrating Wire Gage Positions for the Long Term Monitoring.

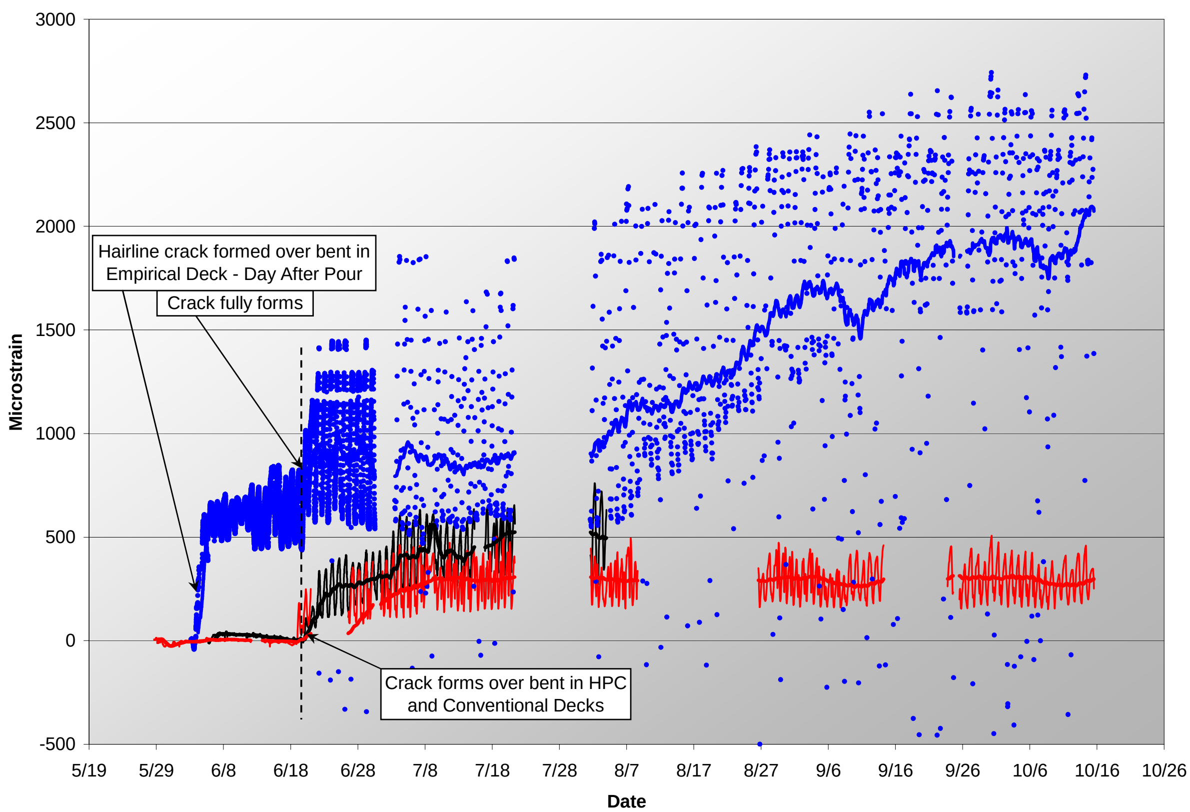

Figure 13: Long Term Data Showing the Formation of Cracks in All Three Decks.

Unlike sensors spanning the bents, sensors installed away from the bents (i.e., position D-3, halfway between the bents and the diaphragms and halfway between two girders) did not show that cracking had occurred. Figure 14 shows the response from vibrating wire strain gages near the top of the deck in each of the decks at this position. Diurnal strain fluctuations are small (~20-70με), indicating contraction due to temperature changes and not cracks in the concrete.

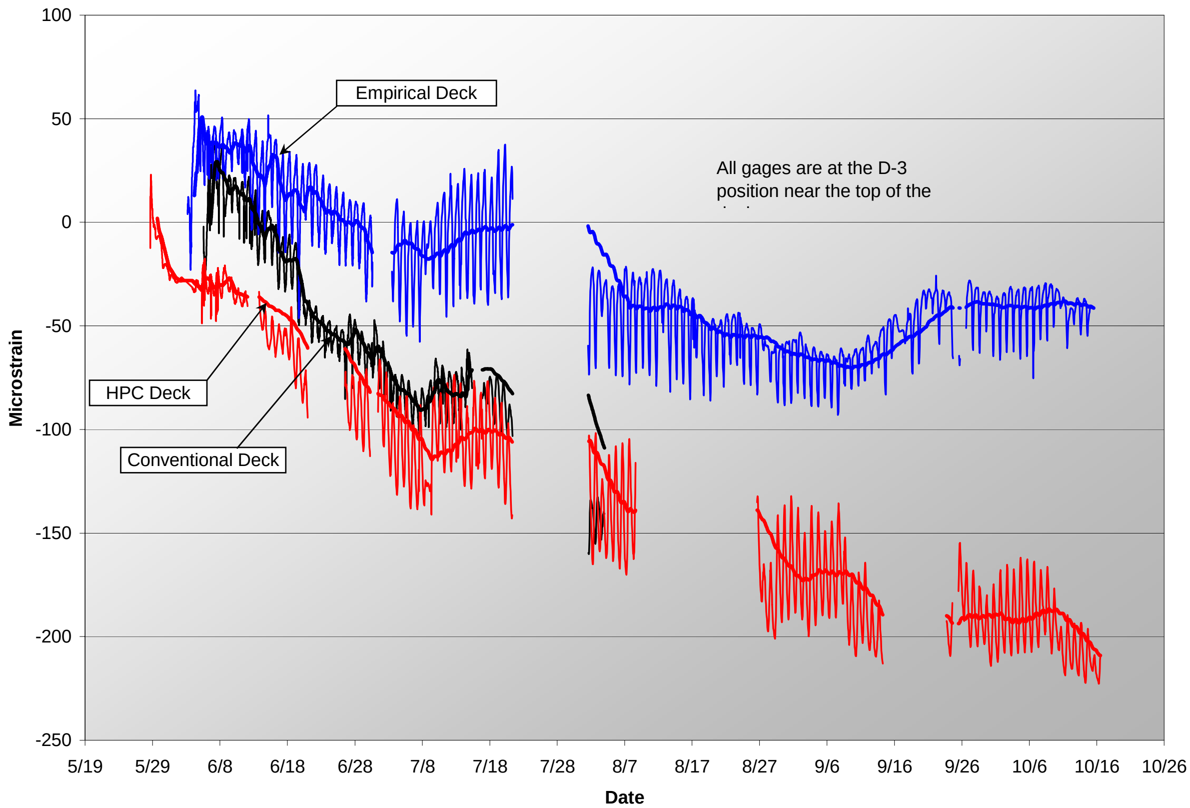

Figure 14: Long Term Data Illustrating Non-Cracked Portion of the Deck.

The formation and presence of a crack can clearly be seen in the data from the Conventional deck presented in Figure 15. In this case, longitudinal strains collected over the interior bent (position F-3) and over the intermediate diaphragm (position B-3) are compared. Prior to cracking, strains in these two locations are almost identical, simultaneously fluctuating based on temperature oscillations. Notably, prior to a crack formation, lower temperatures caused decreases in strain indicating thermal compression. Conversely, higher temperatures produced increases in strain (Line A, Figure 15).

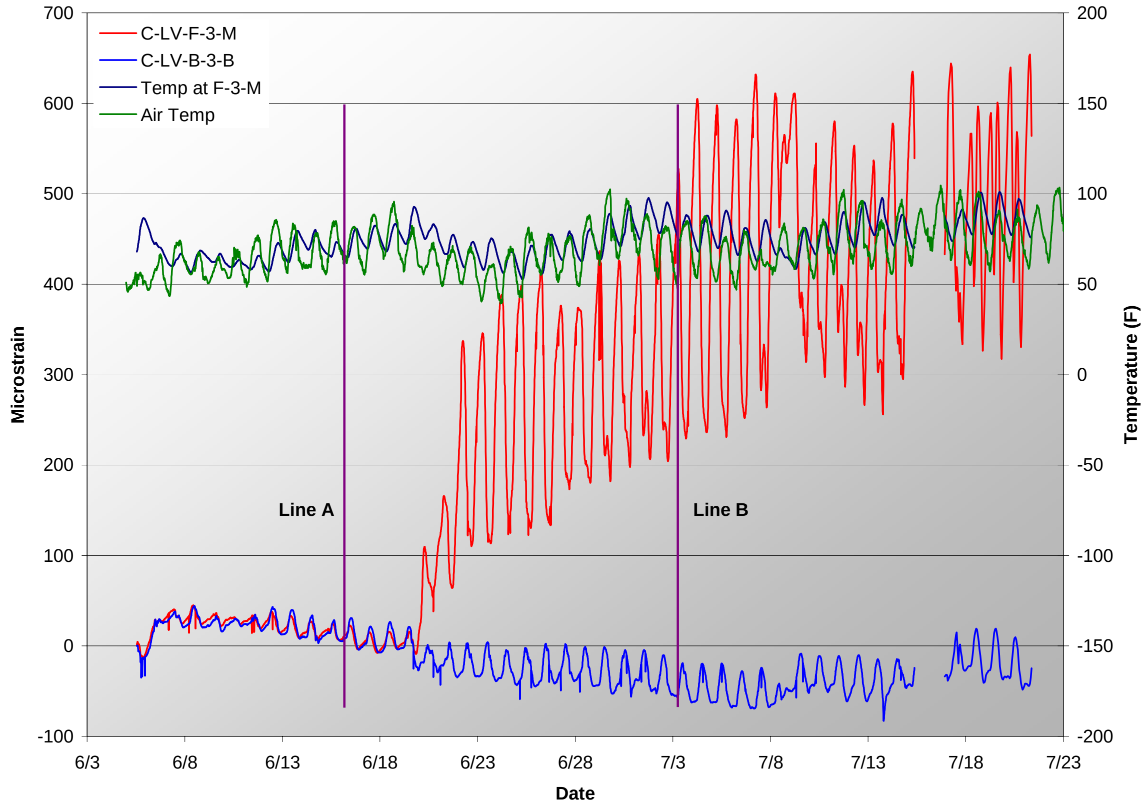

Figure 15: Comparison of Cracked and Non-cracked Position in the Conventional Deck.

On the evening of June 19th, strains over Bent #2 (position F-3) noticeably begin to deviate from those at position B-3. This deviation is most certainly because a crack formed over the bent and intercepted the vibrating wire gage installed at that location. Following the formation of the crack at position F-3, two effects can be observed, 1) diurnal strain fluctuations increase in magnitude at the crack location, and 2) the mean level of strain at the crack location increases in magnitude and becomes tensile in nature due to deck concrete shrinkage. First, daily changes in temperature produce higher strain values over the bent since the bridge deck concrete is essentially separated at that point. This separation causes contractive strains from each section of deck to be realized as increased strains across the crack. Therefore, what used to be decreasing strains over the bents before the crack occurred are now increasing strains. Moreover, the lowest temperatures during the day are now producing the highest strains over the bents (Line B, Figure 15). Strains at position B-3, however, have retained their original cyclic behavior.

Shrinkage of the deck concrete can also be observed using these same two locations. Average strain at both positions is decreasing slightly over time until the crack forms over the bent. At that time, the average strain at position B-3 continues to decrease, but average strain at position F-3 begins to increase. In the body of the deck (B-3), the concrete is shrinking, placing the gage in compression. Conversely, as the “slabs” on either side of the crack shrink, they draw away from the crack, placing the gage at the crack in tension. Possible reasons for the higher rate of increase of the mean strains at the bents (F-3) relative to the body of the slab (B-3) are that 1) strains across the crack represent a concentration of the shrinkage strain effects that are distributed across the adjacent "slabs", and/or 2) ratcheting of the crack is occurring as small concrete particles hold it increasing open after each cycle of expansion.

Monitoring Global Bridge Movements through Surveying

Global bridge movements are to be monitored by measuring relative changes in the elevation and the horizontal position of various points on the surface of each deck. Initial elevation and position measurements were made just prior to conducting the live load tests and were referenced from permanent points installed at the west side of the north abutments of each bridge during construction. Elevation measurements were made at 50 locations on each deck, namely, over the abutments, interior bents, and diaphragms of each bridge at the location of each stringer and between stringers in the instrumented areas of the decks. In the horizontal plane, position measurements were made over the abutments, interior bents, and diaphragms of each bridge at the location of the exterior stringers. Elevation and position measurements also were made at the east and west ends of each abutment. Elevation measurements were shot with an accuracy of approximately 1 millimeter whereas the horizontal locations were measured with an accuracy of approximately 3 millimeters. This information has been input into a database for future reference.

Corrosion Testing

Half-cell potential tests were conducted using the wire leads that were connected to two transverse and two longitudinal bars in each deck. Information from these tests will be summarized and included in the interim report. Preparations were made to gather benchmark carbonation levels. These tests will be conducted next quarter.

Action Items for Next Quarter:

- Conduct carbonation tests

Crack Mapping

The first crack mapping exercise was conducted by Craig Abernathy of MDT Research during the fourth week in July. The results showed that hairline cracks had formed over the bents on all the bridges with the exception of Bent #2 on the HPC deck. Other damage to the decks was also observed, notably that the north edge of the HPC deck was chipped from construction equipment.

Action Items for Next Quarter:

- Conduct 6-month crack mapping and delamination survey (early December)

Task H: Project Reporting

An interim report will be written to summarize all of the activities and data collected as part of this research through December 2003. This report will be delivered to MDT in January of 2004.

Action Items for Next Quarter:

- Quarterly progress report for second quarter for state fiscal year 2004
- Begin compiling information to be included in the interim report due January 2004